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# PROCEEDINGS

## AMERICAN SOCIETY OF CIVIL ENGINEERS

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## DISCUSSION OF PROCEEDINGS - SEPARATES

385, 445, 475, 476

SOIL MECHANICS AND FOUNDATIONS DIVISION

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Discussion of
"PILE FOUNDATIONS FOR BUILDINGS"

by John W. Dunham
(Proc. Sep. 385)

JOHN W. DUNHAM,[1] M. ASCE—The author has studied the discussion of his paper as objectively as he is capable of doing. As was to be expected, some worth-while improvements in the method have been suggested.

It is felt advantageous to take the comments up in the order in which they apply to the paper.

Consultations with specialists on soil problems as suggested by Mr. Focht is a wholesome practice. Structural Engineers cannot, however, avoid responsibility for the design of such an important structural component as a foundation simply because some of the materials involved are soils.

Apparently the intent of the writer's 3rd condition for a successful pile foundation was not clear to Mr. Focht. The intent is simply that at a surface defined by the pile points, all of the load from the piles must be carried by the soil with a suitable margin of safety.

Mr. Focht is entirely correct in stating that the assumptions are the real basis of design. For design purposes the question is, "Will a structure designed to perform satisfactorily under the assumed conditions perform satisfactorily under the actual conditions and be economical?" The following comments are with respect to Mr. Focht's discussion of the assumptions:

1. In example A, the soil above the 27 foot depth was used for support. The results of the second trial were rejected because a longer pile resulted. The possibility that the loose sand in example B will improve during driving has proved to be very real. It suggests the desirability of duplicate borings adjacent to a test pile before and after driving.
2. Mr. Focht gives a very good reason for the second assumption when he writes, "It is often the case that soft clays develop their maximum strength at strains greater than the more stiff clays." It is interesting to note in Mr. Focht's reference No. 7 that a cohesion of .33 tons per square foot was used for clays in the preliminary design of the foundation. Boring M-37 showed a driving resistance on the split-spoon sampler in clay of 5 to 12 blows per foot. The writer's method would use a shear value based upon 5 blows per foot (weakest included stratum) of .33 tons per sq. ft.
3. Mr. Focht s suggestion with respect to the third assumption appears to have merit especially with respect to non-cohesive soils. Assuming that a factor of safety still applies, the value of p for clay, assuming $\phi = o$ would not be significantly different than the writer proposes under "Possible Refinements." Under certain conditions, however, the value of p in non-cohesive soils would be importantly greater than $\frac{N}{5}$ and justifiably so.

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4. Both Mr. Chu and Mr. Focht question this assumption. The idea that the shear strength of the soil is critical has been expressed by A. E. Cummings[2] and accepted by F. J. Converse in Mr. Focht's reference No. 9.
5. Mr. Focht's statement, " Further, the measurement of skin friction with depth in load tests indicate that shear stress does vary as the shear strength," is incompatible with his statement quoted in connection with the second assumption.

Mr. Teng proposes a formula 2A for piles in non-cohesive soils to take advantage of the increase of lateral pressure with the depth of the supporting layer. The suggestion is well worth-while. The writer prefers to write the equation thus:

$$h = \sqrt{(1.58H_1 + H_2)^2 + \frac{J}{o}(P - pa)} - (1.58H_1 + H_2) \qquad (2B)$$

Both Mr. Kantey and Mr. Focht feel that more reliable methods of obtaining soil properties than the " penetration method" are desirable. " N" does not appear in the theory of the writer's method. It is introduced later as one way of evaluating the soil properties required for design. The writer is aware that it does not define those properties as precisely as would be desirable. Tests on undisturbed samples would give more precise values at the points they were taken. Tests in-situ as suggested by Mr. Kantey should be still better. However, the continual and erratic variation of soil properties that is usual throughout the volume of soil supporting a building means that even precise soil properties at points five feet apart vertically and fifty to one hundred feet apart horizontally cannot define the characteristics of the whole foundation with great precision. In selecting the means of evaluating the soil properties in a given case, one should be guided by the importance of the foundation and the quality of the collateral information available, such as experience with nearby structures.

Mr. Chu feels that the friction or shear values derived by the writer's method are excessive. Attention is called to the test cited by Mr. McNulty which seems to require a shear value of over 5 ksf in a mixed soil having an N, prior to driving, of 16 or less.

Simplification of the formulas as suggested by Mr. Chu should help to make them more easily remembered. The rounding off of values appears well within the precision that can be expected in such problems.

The selection of the depth of non-supporting soil is not a matter of judgment but one of finding that depth which will give the shortest possible pile.

A correlation of the writer's method with load tests is greatly to be desired. Since few piles are used singly under buildings, the tests would need to be on pile groups and be carried to failure, to be of much value. The writer does not have available reports of load tests on pile groups that include all of the data required by his method. Such tests are very expensive, which accounts for their scarcity. Masters reported some tests of single piles and piles in groups[3] Unfortunately, no specific soil properties were given. By assuming the soil uniform and deriving the properties from the

2. A. E. Cummings, " Pile Foundations," Purdue Conference on Soil Mechanics and its Applications, Sept. 1940.
3. Masters, F. M. " Timber Friction Pile Foundations." Trans. ASCE, Vol. 108, 1943.

results of the load tests on single piles, the writer was able to compute the capacity of the groups with reasonable success. The reasoning seemed too tenuous, however, to justify its inclusion in the paper. Four piles tested singly at the General Accounting Office project carried 240 kips apiece (the greatest load available) without any sharp break in the load-settlement curve and a maximum gross settlement of less than $\frac{1}{2}$ inch. The writer's method indicated that the safe loads as isolated piles, neglecting the taper ranged from 72 kips to 100 kips. This tells little about the capacity of groups of such piles, however, since shear between the piles and the soil was critical for the isolated piles, whereas bearing below the pile points would be critical for the large groups to be used. In groups, the safe loads for these piles, by the writer's method, were 43 to 70 kips per pile.

Mr. McNulty seems to reject completely any consideration of driving resistance. Regardless of how the lengths of piles are established at the several points where the designer feels he has enough information to make the decision, a criterion is required for the lengths of piles between those points. The writer uses a maximum point elevation and driving resistance for that purpose.

The proposed method yields more conservative results than seem justified in the circumstances cited by Mr. McNulty. There are two considerations that may contribute to the disparity.

The many ambiguities and uncertainties inherent, at best, in foundation design justify a policy of resolving practically all uncertainties on the side of safety. Such a policy must be expected to result occasionally in an excessively conservative design.

Another possible contributing cause is that the soil was materially changed for the better in the pile driving operation. The heavy taper of the Raymond Standard Pile would tend to increase such an effect. This possibility and a way of evaluating it and dealing with it has been discussed earlier in connection with Mr. Focht's comments.

Corrections for Transactions.—The expression at the end of the second sentence following Eq. 12 should be $\sqrt{\frac{P - 0.20Na}{0.0315Ko}}$. The expression at the end of the third sentence should be $2\sqrt{\frac{P - 0.20Na}{0.0315Ko}} - .58H_1$.

The author is grateful to the gentlemen who have taken the trouble to discuss his paper. He still feels that the method he has presented, modified as suggested by the discussion and indicated by further experience, and adjusted as necessary to fit unforeseen circumstances, is a sound and workable approach to the design of pile foundations for buildings.

Discussion of
"SUCTION FORCES IN SOILS UPON FREEZING"

by Alfreds R. Jumikis
(Proc. Sep. 445)

ROBERT RUCKLI [1]—1. Definition of Equation of Suction Force—In describing the problem of suction force in soils upon freezing, Mr. Jumikis introduces in the definition of the equation of suction force a "coefficient of thermo-osmotic permeability $k_S$" as a substitute for the Darcy's coefficient of permeability, $k_D$. According to the definition for $k_S$, this new coefficient should have the dimension of $[(cm)^2 \cdot (sec)^{-1} \cdot (^oC)^{-1}]$. Introducing it in Eq. (9), one obtains for the suction head, $h_S$, the dimension of (°C). This does not agree with the definition of suction which has a linear dimension (cm). The water transport due to the growth of ice lenses is controlled by Darcy's law and not by the thermo-osmotic phenomenon where, according to Winterkorn, the movement of water takes place only in the films surrounding the pores. In applying the Darcy's law it is immaterial whether the hydraulic gradient (i) is caused by gravity or any other force. According to the writer's belief and the mathematical definition of the suction force, the formation of ice lenses causes, by molecular action, in the water pores lying underneath, a depression which spreads out in the deeper layers of the soil and causes a flow from the ground-water toward the ice lenses.

2. Suction Measurement—The contents of the paper under discussion remind the writer of a test performed in Switzerland several years ago on measuring the suction force underneath ice lenses, and about which the writer reported in the Proceedings of the 2nd International Conference on Soil Mechanics and Foundation Engineering, Vol. II, p. 282, Rotterdam, 1948. It should be said here that these tests were interrupted for several years, but it is hoped that they will be resumed and continued in the Soil Mechanics Laboratory of the Swiss Federal Institute of Technology in Zürich. The purpose of the tests is to learn whether or not suction force can be considered a soil constant, like, for example, permeability or capillarity. It seems that it is extremely difficult to measure the sub-pressure. Therefore it is of great interest to learn about such work being done by research workers elsewhere.

3. Criterion for the Frost Sensitivity of Soils—Assuming that the upward flow from the ground-water toward the ice lenses is really caused by the suction force $h_S = \frac{P}{\gamma}$, it can be shown that for one-, two- and three-dimensional flow the amount of water moved is proportional to the product $(k_D \cdot h_S)$. This product can therefore be considered a criterion for the frost sensitivity of a soil. Tests performed in Zürich and elsewhere have shown that it is not possible to trace an exact limit between frost-heaving and non-frost-heaving soils or aggregates merely on a granulometric basis. Soils,

1. Chf., Road-Construction Section, Swiss Federal Inspectorate, Bern, Switzerland.

even those which satisfy the famous criterion by A. Casagrande, can, under favorable conditions, show ice segregation. In Switzerland, this fact receives close attention, particularly when one has to decide whether base course material from gravel pits can be tolerated or should be rejected. The experience of many years indicates that well-graded aggregates containing more than 3% of particles finer than 0.02 mm do not cause damage to roads by frost, regardless of the fact that the subsoil and the climatic conditions were favorable for the formation of ice lenses. This leads one to consider not only the technological composition of the aggregates but also the hydrological and geotechnical condition of the site in question. Therefore, it seems that for the better understanding of the suction phenomenon and for the evaluation of soil material, the concept of the suction force could, probably, be a useful aid.

Besides, as the writer has demonstrated in his book "Der Frost im Baugrund" (The Frost in Soil), published by Springer, Vienna, 1950, by applying the concept of suction force it is also possible to examine the similarity (model law) existing between freezing soil samples of different sizes.

In conclusion, the writer wishes to express his pleasure at the opportunity afforded him to participate in this discussion and to learn from the presented paper that the difficult subject of suction force in soils upon freezing is receiving attention in America. Frost is, naturally, a matter of great concern to the Swiss Alpine regions, and therefore the work being done abroad is of considerable interest to Swiss engineers from both a theoretical and a practical point of view.

HANS F. WINTERKORN,[2] A.M. ASCE—The problem of frost action and damage in highways and their foundations is a result of the interaction of heat and moisture transmission under existing thermal and other energy potentials and of the peculiar physico-chemical properties of the water substance. The fact that the involved potentials are themselves periodic functions of time increases the difficulty of mathematical analysis already of the component phenomena to such an extent that it exceeds the normal mathematical equipment of the highway engineer. Even the expert mathematical physicist would find it difficult, if not impossible, to carry through a rigorous analysis of the entire problem complex, assuming that all the physical factors and parameters were known.

In the partial scientific and in the practical engineering solutions of the problem which have been accomplished so far, mathematical simplifications and physical assumptions have been introduced to such an extent and in such a way that it is often difficult to decide whether an assumption introduced for mathematical reasons can be justified from a physical point of view. Under these conditions it is definitely important to isolate and clarify the physical phenomena and concepts involved in the problem. Recognizing this need, Dr. Jumikis has given his attention to the suction force associated with the formation of ice lenses in soils, after having demonstrated his understanding and mastery of the general problem in his excellent previous report on "Theoretical Treatment of the Frost Penetration Problem in Highway Engineering."[3]

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2. Associate Prof. of Civ. Eng., School of Engineering, Princeton Univ., Princeton, N. J.
3. Engineering Experiment Station, University of Delaware, Newark, Delaware, May 7, 1952.

In the present paper, Dr. Jumikis has correctly identified the water transmission to the freezing zone as being of the nature of "thermo-osmosis" and he has given a very apt physical picture of the entire process. However, in view of the fact that the general phenomenon of thermo-osmosis has as yet not been completely elucidated, the criticism may be ventured by some that the suction problem has not been solved but has just been given another name. Fortunately, this is not the case at least for the simplified conditions depicted in Fig. 1 of the paper by Dr. Jumikis. According to this figure an ice lens is growing at the distance "1" from the ground water level. The water temperature at the ground water level is $T_2$, that of the growing ice lens is $T_1$. The ice lens and the ground water level are connected by a conducting system. With the two limiting energy conditions "liquid water at temperature $T_2$" and "ice at temperature $T_1$" established, and only postulating the continuity of the conducting system, one can calculate the difference in heat content of a certain volume of water at these two conditions. This difference in heat content equals the amount of heat energy conducted in the process from the higher to the lower temperature. If this conduction is sufficiently slow to be reversible at any stage, one may calculate the free energy or maximum work available for conductance of the water, since the transfer of the heat energy is associated with the water transfer. According to the second law of thermodynamics:

$$W_{max} = Q \frac{(T_2 - T_1)}{T_2} \qquad (1)$$

in which $W_{max}$ = free energy or maximum work

$Q$ = quantity of heat energy transferred from $T_2$ to $T_1$ including heat of fusion;

$T_1$ and $T_2$, the ground water and ice temperatures respectively.

Q is composed of heat of fusion "q" and the difference in heat content of the transferred material at the two limiting temperatures, without change in phase. Because "q" is quite large in comparison with the second factor, the latter is neglected in the following approximate derivation of the suction pressure.

The quotient of a free energy divided by the volume with which it is associated has the properties of a pressure or suction. Using the metric system and availing oneself of the advantage that in this system the weight of one cc of water and approximately of ice is one gram, and taking the heat of fusion of one g of water as 79.7 cal, one obtains for the suction "P":

$$P_{max} = \frac{W_{max}}{1\ cc} = \frac{79.7\ cal}{1\ cc} \frac{(T_2 - T_1)}{T_2} \qquad (2)$$

Since: 1 cal = 42,660 g cm and

$$79.7\ cal = 3.4 \times 10^6\ g\ cm$$

$$P = 3.4 \times 10^6 \frac{(T_1 - T_2)}{T_2} \qquad (3)$$

P is the difference in pressure, due to the work function between the ground water level and the freezing ice lens. If the temperature varies proportionally with depth one may write:

$$\text{pressure gradient} = i = \frac{P}{1} = \frac{3.4 \times 10^6\ (T_1 - T_2)}{1.T_2} \qquad (4)$$

If in such an extremely simplified system there were the numerical values of:

$$T_2 = 283^\circ \text{ K}$$

$$T_1 = 273^\circ \text{ K}$$

$$h = 300 \text{ cm}$$

one would obtain:

$$i = \frac{3.4 \times 10^6 \times 10}{283 \times 300} = \frac{3.4 \times 10^7}{8.5 \times 10^4} = 400 \text{ cm head/cm.}$$

This suction or negative pressure can be introduced into any formula of the D'Arcy type irrespective of the exact nature of the flow. This means that it is equally applicable for saturated and for unsaturated capillary flow.

If the temperature gradient varies with depth, then the differential equation of the second law of thermodynamics must be used. Accordingly, for the region proximal to the ice lens:

$$P_{max} = \frac{W_{max}}{1 \text{ cc}} = 3.4 \times 10^6 \frac{dT}{T} = \frac{3.4 \times 10^6 \, dT}{273} = 1.25 \cdot 10^4 \, dT.$$

If proximal to the growing ice lens, the temperature gradient is $\frac{dT}{dx}$, then the corresponding "suction" gradient is:

$$i = 1.25 \times 10^4 \times \frac{dT}{dx} \qquad (5)$$

These suction values are, of course, maximum theoretical values attained under the assumption that the thermodynamic system is working with a 100% efficiency. It is very important for practical and theoretical reasons to obtain experimental suction values, and the writer is looking forward with great interest to the results of the present experiments of Dr. Jumikis.

So far the writer has treated the problem of ice lens formation as essentially one of increasing the entropy of the system in accordance with the second law of thermodynamics. However, the problem has other physical aspects. If one considers the ice formation simply as the cause of the suction force assuming that the water movement itself follows D'Arcy's law, one can use a coefficient of permeability determined by water movement under hydrostatic pressure and write for the volume "v" of water transported per second through a cross section of 1 $cm^2$:

$$v = k.i = k.1.25 \times 10^4 \frac{dt}{dx} \qquad (6)$$

where k is the permeability coefficient and i the pressure or suction gradient.

The latent heat of freezing for one gram of water is approximately 80 gram-calories. Of this, $\frac{dT}{T} \times 80$ has been used for transportation leaving $80(1 - \frac{dT}{T})$ for heat transfer. Assuming $\frac{dT}{T}$ small enough to be neglected and using "v" in its previous defined meaning, one may write for the heat transfer per second per sq. cm. resulting from the suction potential:

$$\text{heat transfer} = v.80 \text{ cal} = k.1.25.10^4 \frac{dT}{dx} \cdot 80 \text{ cal/sec. cm}^2. \qquad (7)$$

We are interested in the value of the permeability coefficient at which this process of heat transfer becomes more efficient than that of pure heat

conduction. For the latter, one may write:

$$Q = c \cdot \frac{dt}{dx}$$ in c.g.s. units; taking as a plausible value:

$c = 2.5 \times 10^{-3}$ in ca./sec cm$^2$ per $^0$t/cm; at equal effectiveness of the two processes:

$$k \times 1.25 \times 80 \times 10^4 \frac{dt}{dx} = 2.5 \times 10^{-3} \frac{dt}{dx}$$

hence: $k_{lim.} = \frac{2.5 \times 10^{-3}}{1 \times 10^6} = 2.5 \times 10^{-9}$cm sec.$^{-1}$

Consequently the advantage of lowering the entropy of the system by moisture flow to the ice lens over plain heat conduction extends to permeability values as low as $10^{-9}$. In systems possessing very low permeabilities and in cases where the ground water level is at a considerable distance, ice lens formation results in drying out the surrounding soil. This may go to a point at which simple heat conduction is a more efficient heat transfer mechanism than moisture flow under a suction potential. Consequently, the freezing temperature penetrates to the lower layers until a location is reached where moisture flow again becomes more efficient than plain heat conduction with respect to the lowering of the entropy. As pointed out elsewhere,[4] this phenomenon is theoretically akin to the rhythmic precipitation that is responsible for the colored bands in agates, sandstones and other mineral systems.

The writer wants to express his appreciation of having this opportunity to discuss the excellent work of Dr. Jumikis. This discussion has been kept as simple as possible in order to bring out some physical points that were considered important. In concluding, regret is expressed that limitation in time available at the meeting permitted Dr. Jumikis to present only a small portion of his most important work.

4. Principles and Practice of Soil Stabilization. " Colloid Chemistry, Theoretical and Applied" (Ed. J. Alexander) VI, Reinhold Publishing Corporation. 1946.

Discussion of
"SAFE LOADS ON DOG-LEG PILES"

by James D. Parsons and Stanley D. Wilson
(Proc. Sep. 475)

A. E. CUMMINGS,[1] M. ASCE—This paper is concerned with a subject in which the writer has been interested for many years. As the authors have remarked, many engineers give little or no thought to the probable alignment of long wood piles or H-beam piles after they are driven into the ground. Regardless of the fact that no inspection of such piles can be made, they are often allowed to carry substantial loads. When long pipe piles and especially the combination pipe and shell pile came into general use, it was soon observed by internal inspection of the driven piles that they were not always perfectly plumb and perfectly straight. During driving the piles were sometimes distorted into long pitch helical curves, sometimes into long sweeping bends and now and then into sharper bends such as the authors describe as "dog-legs."

As soon as it became known that such misalignments were occurring, field investigations of all kinds were undertaken to determine whether or not the misalignment affected the carrying capacity of the pile and, if so, to what extent. Slope indicators similar to that described by the authors have been in use in the oil well drilling industry for years. Some of these tools can determine not only the amount of slope but also its direction with respect to the compass. Equipment of this kind has been used to determine the alignment of driven piles. Also, many field loading tests have been made on piles with various amounts of misalignment in various soil conditions. The results are always similar to those shown in Figure 2. The piles which were out of alignment carried the test loads and rebounded when unloaded just as satisfactorily as did piles which were straight and vertical.

In the writer's opinion, the satisfactory behavior of piles which are not in perfect alignment is due largely to a factor which the authors have neither mentioned nor taken into account in their theoretical analysis. They have discussed the ability of the surrounding soil to stabilize the pile against horizontal displacement but have said nothing about the interaction between pile and soil in the vertical direction. When an external vertical load is applied to the head of a pile, this load is distributed in various ways to the soil in which the pile is embedded. It is customary to refer to piles as "friction" piles or "point bearing" piles. All actual piles are combinations of these two types since some of the load goes into the soil by friction along the pile shaft and the balance is delivered to the soil at the point of the pile.

Numerous full-scale field tests have been made to investigate the manner in which piles distribute their loads to the soil in which they are embedded. (4) (5)[2] The most recent tests of this type are those made by the Nebraska Department of Roads and Irrigation at Omaha (6) and those made by the Corps

1. Director of Research, Raymond Concrete Pile Co., New York, N. Y.
2. References (4)(5)(6)(7) are listed at the end of this paper.

of Engineers of the U. S. Army at the West Atchafalaya Floodway in Louisiana (7). The tests described in these publications were made on several kinds of piles in various soil conditions. On piles that were not driven to point bearing on hard ground practically all of the load was delivered to the soil along the pile shaft and almost none of it reached the point of the pile. Even when a long pile was driven through a deep bed of soft soil to high point resistance in hard ground only a small percentage of the test load reached the pile point. The results of such a test made by the Corps of Engineers in Louisiana are shown in Figure A.

The pile was a hollow tapered pile made of No. 3 gage steel plate. It had a point diameter of 8 inches and a butt diameter of 16 inches. It was driven with a single acting steam hammer having a 5000 pound ram and a 3 foot stroke. The soil consisted of clays and silts to a depth of about 85 feet below the ground surface where a bed of compact sand was encountered. As can be seen from the driving record, the resistance built up gradually to about 45 blows per foot when the pile point was about to reach the sand stratum. Within a few feet the resistance increased rapidly to about 100 blows per foot, then to 435 blows per foot and driving was stopped at a final resistance of 100 blows for the last ½ inch with the pile point about 90 feet below the ground surface.

Before the pile was driven, SR-4 strain gages were attached to the inner surface of the tube at vertical intervals of about 5 feet. The installation and operation of the strain gages was done by the Office of the Research Engineer of the Association of American Railroads. When the static test load was applied, strain gage readings were made at each load increment to determine how much of the load was in the pile shaft at various levels. The results of some of these readings are shown in Figure B. When the test load amounted to 140 kips, most of the load was transmitted to the soil along the upper half of the pile shaft and only about 20 kips reached the point. Even at the maximum load of 300 kips, only about 50 kips reached the point of the pile. The testing of this pile occupied a period of 5 days and the pile carried the maximum test load for 24 hours without settlement. Similar results have been obtained in all of the tests that have ever been made to investigate the transmission of load from pile to soil.

In the force diagrams of Figures 10 and 12, it has been assumed that all of the load at the top of the pile is transmitted undiminished to the pile point. The calculation of stresses in the pile was made on the basis of this assumption. In both cases, the embedded surface of the pile below the level of the filled ground amounted to about 250 square feet. An average frictional resistance of 375 pounds per square foot would support the entire 94 kips of Figure 10 and only 295 pounds per square foot would be required to support the 74 kips of Figure 12. From such considerations as well as from the results of the field test referred to above, it is evident that only a fraction of the load applied to the pile head could have reached the pile point. Calculations of bending moments and stresses in the pile based on the full point bearing assumption are therefore extremely conservative. Even when the pile has been driven to hard ground through saturated plastic soils subject to consolidation, it need not be expected that a pile which is not in perfect alignment will behave much differently from a pile which is perfectly straight and plumb.

The authors have made a valuable contribution to this whole subject by showing that piles should not be abandoned simply because the bottom of the pile cannot be seen from the top. However, it is the writer's opinion that it

is not necessary to reduce the allowable load on the pile whenever the results of a load test are as good as those observed on Piles 15-4 and 59-2. The calculations of the stresses in the bent pile is a statically indeterminate problem involving unknown interactions between the pile and the soil both vertically and horizontally. The determination of the allowable load should therefore be based on the results of a load test rather than on a theoretical analysis or an arbitrary building code regulation about straightness. All of the available evidence indicates that a curved or bent pile is not in danger of collapsing or buckling even when it is loaded well above the normal working load.

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6. Henry G. Schlitt, " Group Pile Loads in Plastic Soils." Proc. Highway Research Board, 1952, Vol. 31, p. 62.

7. AREA Bulletin 489, September-October 1950. Vol. 52, No. 489, page 149.

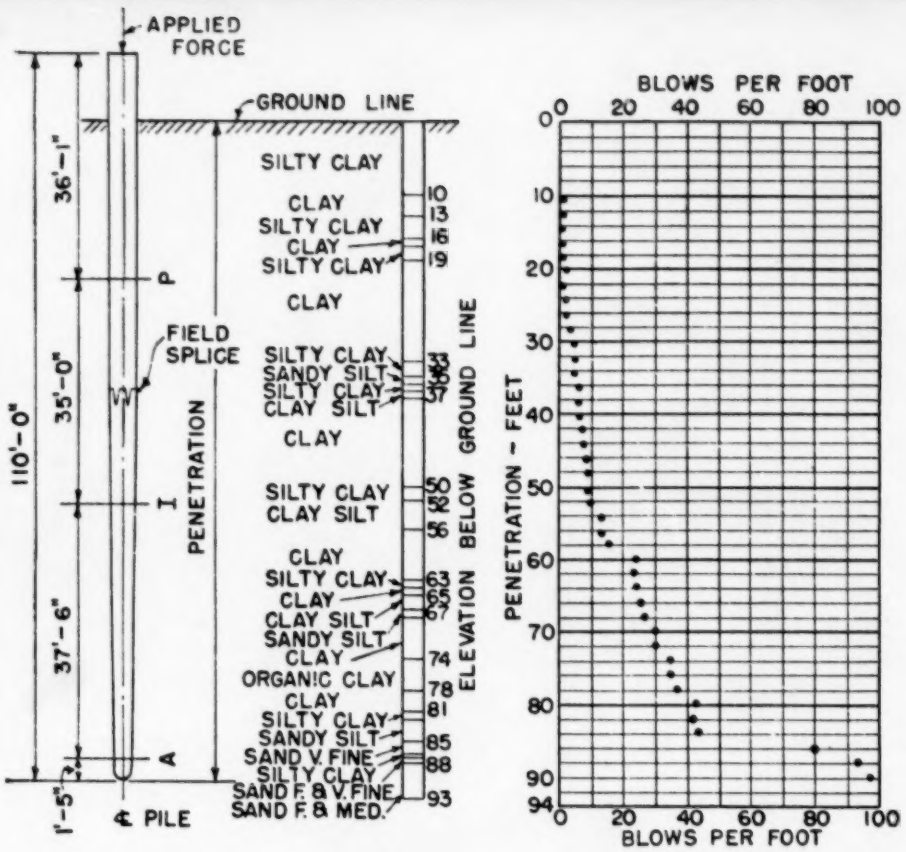


Fig. A LOG OF BORINGS DRIVING RECORD

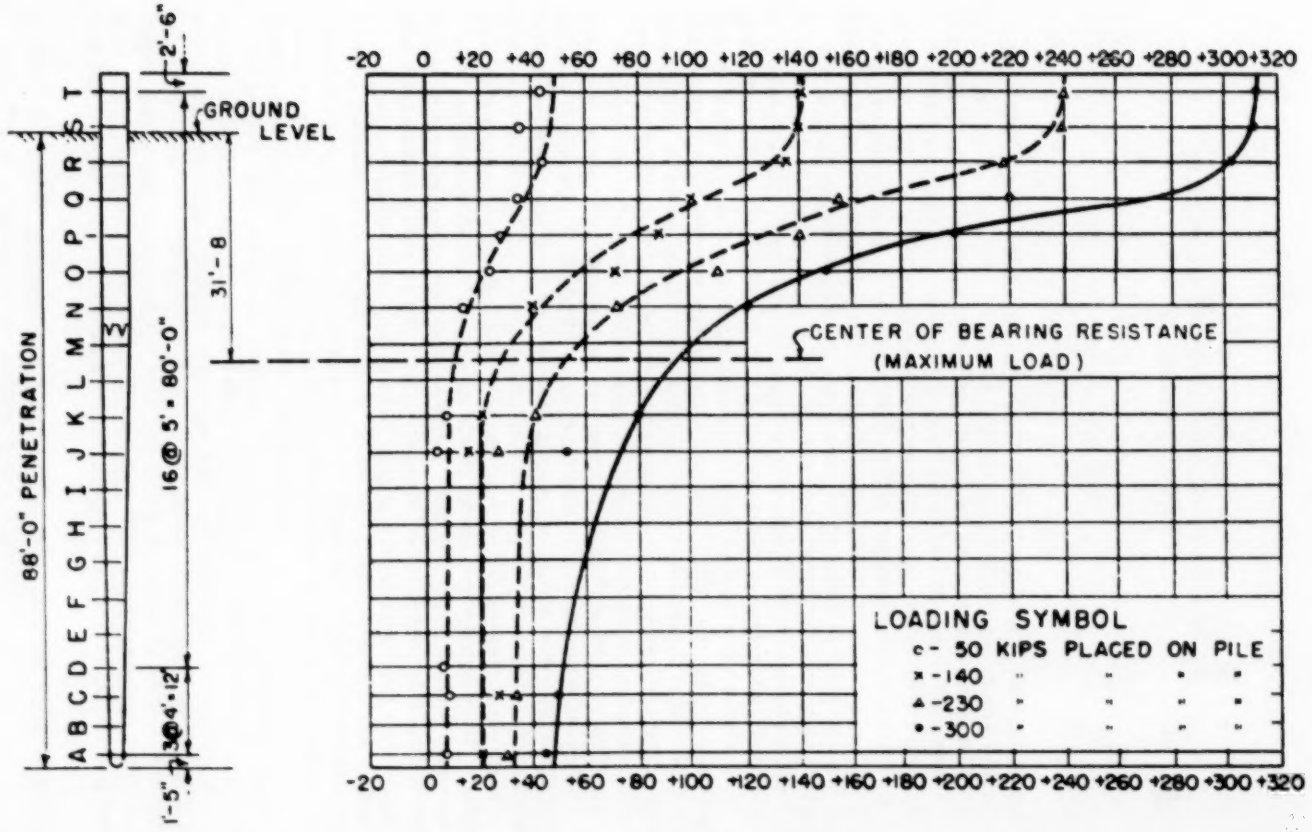
2'-6"
GROUND LEVEL
31'-8
16 @ 5' = 80'-0"
4'-12"
1'-5"
88'-0" PENETRATION
A B C D E F G H I J K L M N O P Q R S T
-20 0 +20 +40 +60 +80 +100 +120 +140 +160 +180 +200 +220 +240 +260 +280 +300 +320
CENTER OF BEARING RESISTANCE
(MAXIMUM LOAD)
LOADING SYMBOL
o - 50 KIPS PLACED ON PILE
x - 140 " " " "
Δ - 230 " " " "
• - 300 " " " "

Fig. B PILE REACTIONS AT EACH SECTION - KIPS

Discussion of
"STUDIES OF BEARPAW SHALE AT A DAMSITE IN SASKATCHEWAN"

by Robert Peterson
(Proc. Sep. 476)

J. D. MOLLARD,[1] M.E.I.C., and D. H. POLLOCK,[2]—Mr. Peterson's excellent paper on Bearpaw Shales is particularly significant at this time. Fig. 1 in Peterson's paper shows that these shales and those of similar nature underlie extensive areas of the northern Great Plains region where several large dams are presently being constructed. An examination of slopes along a number of large valleys of this region shows that stability problems associated with these shales are regional.

The large slumps found in this region and discussed by Peterson are a special variety; that is, they do not appear to be duplicated in other geologic environments (1). They are distinguished mainly by the large area that has been affected by slope movement. Along the South Saskatchewan River, these slumps range from $\frac{1}{4}$ to 2 miles in length along the river and extend $\frac{1}{2}$ to more than a mile into the upland from the toe of valley slopes. In addition, they appear to be indigenous to poorly or semi-consolidated, clay-shale bedrock country that has been covered in recent geologic times by glacial drift.

Because the area over which slope failure has occurred is particularly large and subsurface exploration of a number of slide areas, therefore, prohibitive (Peterson, P. 9, L. 26), and because a visual examination of the failure zone is precluded by a mantle of unconsolidated material, critical field observation of the slump areas was considered necessary. In addition, many slump areas exhibit a stable appearance at the present time while others manifestly do not; also large slump areas occur along certain reaches of the valley and not along other reaches. Accordingly it became apparent that information based on the interpretation of natural ground slopes as well as the geology of the slide was needed to supplement data from laboratory tests (Peterson, P. 2, L. 47; P. 3, L. 2 and 5). It is now fairly well established that slope movement in many of the slides is a slow nevertheless continual creep—a type of failure that is difficult to evaluate by laboratory experiment because of the time factor involved.

On page 3 under Regional Geology, Peterson says: "Most of the area shown in Figure 1 has been glaciated with the result that the bedrock is generally covered with a mantle of glacial drift except along the valleys where outcrops occur." The present geologic study indicates that glaciation of this region has much influenced the stability of the Bearpaw Shale and in large part is responsible for its unusual behavior. The intervention of glaciation into the normal cycle of fluvial erosion resulted in fairly rapid loading and

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unloading of the land, followed by rapid drainage trenching through the glacial deposits and into the shale beneath. The net result produced a favorable environment (see below) for the formation of large-scale slope failure.

The effect on slope stability of present-day erosion, which is cutting through overburden materials and into the shale, is commented on by Peterson (P. 11, L. 3): " In the case of gullies [ along which large-scale slumping has occurred ] it is known that shale is near the surface in the bottom." As seen from the following discussion, the position of the shale surface in relation to the bottom of valleys and gullies in this region largely determines present-day slope stability and, accordingly, markedly influences structure location and layout.

The relation of slumps to places of riverbank erosion is also noted by Peterson (P. 10, L. 27): " It would appear that many slides have occurred as a result of river bank erosion." The present study supports this view and, because of its importance in site selection, considers it in some detail below.

As a result of the foregoing points raised by Peterson, the first part of the following discussion deals principally with environmental conditions as they relate to the development of slumps in this region. These are the surface expression and characteristic location of large slide areas and the arrangement of geologic materials found in slide areas. Following this the type, age and rate of slope movement along the South Saskatchewan River and its main tributaries in the vicinity of a proposed damsite are considered.

An airphoto study was made of twenty large areas in the vicinity of a proposed damsite to determine surface features that the slide areas have in common (2). Several surface features strongly suggest that all slides were formed in much the same way, during the same period of time and that very likely they have all been caused by the same set of contributory factors. In fact, several slide areas appear so similar and recurrent in the arrangement of geologic materials and topographic setting that it seems possible to venture a natural expression whose terms, or environmental factors, could be used to predict localities where further movement is likely to develop along the South Saskatchewan and in similar valleys in this general region.

## Geologic Setting in Slide Areas

From a stereo-inspection of vertical aerial photographs and subsequent detailed field checking it appears that the following conditions are necessary before large slumps are produced; moreover, that the absence of any one condition results in slope movement that is negligible, if it can be discerned at all:

1. A deep postglacial valley in which the soft and weathered clay-shale surface is exposed above the bottom of erosion valleys. In the case of the South Saskatchewan River this elevation datum was taken to be the present river level.

2. A mantle of glacial drift deposited upon clay-shales whose buried surface is above the valley bottom (condition 1). These glacial materials range from glacial lake clays, silts and sands to glacial till and glacial outwash sands and gravels. In a few areas much of the glacial drift has been removed by erosion; but here a deep drift mantle is still evident on the adjoining upland.

3. A change in river direction, creating places where lateral stream erosion steepens and undercuts the valley bank for a considerable distance; and at the mouths of deeply incised tributary creeks to the main river,

especially where creek gradients are fairly steep and creekbeds have cut through the glacial drift and into the clay-shale (conditions 1 and 2, above).

## Evidence Supporting the Requirements Listed Above

The following airphoto and field observations support these three natural requirements for the production of large-scale slumps in this region:

1. Each of the twenty slump areas visited showed soft and weathered shale exposed either along gullies cut into the slumps or along the main river bank. On the other hand, drillholes extending through glacial drift and into the shale at three rather extensive stable areas along the valley sides (yet adjacent large slide areas) revealed that the surface of the shale at the stable areas was below the level of the valley bottom for a distance of several hundred feet back from the toe of valley slope. In a number of large valleys in this region, similar in many respects to the South Saskatchewan River Valley, valley slopes show no indication of having moved where the entire slope-forming materials are composed of glacial materials to a depth below the valley bottom, even though these slopes are in many cases both steeper and higher than those found in the slump areas along the South Saskatchewan (see Fig. 1).

At one cutbank location along the South Saskatchewan River in the vicinity of the proposed damsite, a stable 100-foot-high, near-vertical bluff of dense glacial till extending below river level overlooks the present river level. Yet on flatter slopes valley sides have slumped in nearby areas where the drift-covered, weathered shale outcrops above river level. Another way of stating the above is: When the shale profile (i.e., surface) lies below river level the valley sides are stable; a shale profile above river level, on the other hand, creates a potentially unstable slope.

2. At all slump areas the upland is covered with a variable depth of glacial drift. Indeed, glacial materials were observed to cover clay-shale at all slumps visited in the field, though at one notable instance much of the drift has been eroded away. Usually the depth of glacial debris varied from 40 to 200 feet.

In several areas where thick, uniform clay-shales (i.e., the Bearpaw and similar types of clay-shales observed elsewhere) have developed highly dissected relief as a result of long-continued subaerial erosion, under natural conditions large slides were not observed on aerial mosaics. For that matter, very few small slides were noticed. Such areas have been observed in the non-glaciated clay-shale areas of Montana, South Dakota, Maryland, California and Middle East Asia, where slope heights and gradients are comparable to those along the South Saskatchewan River. It is significant that the above localities (viewed in aerial photos) have not been covered by continental glaciers. It would appear that the steep slopes and impervious character of these massive clay-shales causes high run-off, little or no internal seepage and consequently very little loss in shear strength. Layered materials having different permeabilities in adjoining layers, on the other hand, are very susceptible to the formation of slides (1).

In the glaciated regions of the northern Great Plains, underground drainage above and along the shale-glacial drift contact is frequently blocked by impervious glacial materials, causing increased moisture migration through the weathered near-surface zone of the shales and resulting, in time, in a general softening of the material.

A greater propensity for valley side slopes to fail was noticed where thick pervious sediments overlie the Bearpaw shales. The pervious beds permit

percolating ground water to be readily transmitted to the buried shale surface, thereby increasing weathering activity and generally softening the surface shale zone. Also inasmuch as slope movement is largely in a horizontal direction and in the shale, the superimposed granular materials, lacking cohesion, offer little shearing resistance.

3. Fig. 2 of this discussion illustrates the correlation between the location of major slides and places where lateral river erosion has been active in recent times. Nine major slump areas in the vicinity of the proposed damsite have steep cutbanks at the river's edge and lack flood-plain development. The location of these large slide areas is customarily slightly downstream from changes in river direction, places where the sweep of the current has for a long time been directed in toward the river bank. Conversely, nine slip-off slopes in the same reach of the river showed no signs of movement.

At large slump areas near the mouths of tributaries to the main river, downward erosion in creek beds has been especially rapid. A large volume of material has been eroded from these sit ı in a comparatively short period of time. The vertical arrangement of geologic materials at these places is identical to that found at other slide areas along the main river, that is, glacial drift over soft and weathered clay-shale (see Peterson, P. 11, L. 3). Slopes along tributaries and along the main river having the greatest height differential between the upland and streambed level show the largest area affected by movement (i.e., normal to the direction of flow in the creek or river).

## Type, Age and Rate of Slope Movement

Recent detailed investigations in the R-1 area (Fig. 1, Peterson) have provided additional evidence regarding the type, rate and recency of slope movement.

Peterson mentions (P. 4, L. 38 and P. 10, L. 24) the probability of horizontal slope movement. Recent investigations confirm this but also note the qualification that concurrent vertical subsidence of rather large blocks of ground accompany the essentially horizontal slope displacement—especially in areas showing complexly deformed slump topography (Fig. 4).

Peterson mentions (P. 4, L. 41) that the " . . . . dykes raise the possibility of movement having occurred before the last retreat of the ice." He also notes (P. 9, L. 41): " The movement is believed to be very old, possibly having occurred before the retreat of the last ice sheet." Further (P. 16, L. 15): " In most cases no movement is discernible at the present time."

Recent evidence points to the fact that the slump topography observed along the main river and its tributaries is postglacial in age (i.e., after the retreat of the last ice sheet) and that while some of the slump areas are stable at the present time, others are now definitely known to be moving at an imperceptibly slow though measurable rate. Careful field measurements taken on observation hubs in November, 1954 (four years after these hubs were first placed) reveal that local areas in the slump terrain are creeping at the apparent rate of approximately 3 inches per decade (c.f. Peterson P. 9, L. 34).

On page 10, line 29 Peterson states that " . . . it is believed that the slides occurred slowly and intermittently over a period of many hundred years." In view of what has been reported above, the following should perhaps be added to Peterson's statement: " . . . and slow and intermittent movement is still going on today in some sections of the slump areas."

A bulldozer excavation, extending from the upland to the first ridge adjacent the upland in the R-1 area, has disclosed a number of features that help explain the manner and recency of movement. These features are shown on Fig. 3 and are as follows:

(a) Similar orientation of the weathered surface soil (topsoil) at locations 1 and 2 indicate that the ridge experienced very little rotation in its riverward movement.

(b) The failure plane marked 3 is the result of a reduction in lateral earth pressure in the sand and find gravel, the reduction being due to riverward movement in the underlying shale. The dip of the failure plane in the stratified, pervious materials closely approximates the angle obtained from Mohr's diagram, employing appropriate values for $\phi$, the angle of internal friction for this material, and the average, overall slope of the valley side.

(c) The weathered soil surface on the buried, downdropped block at 4 appears equal in leaching, depth and development of profile characteristics to the weathered soil surface that is presently developing on the ground surface at 2. This feature establishes that initial downward movement of mass 4 was very recent inasmuch as there has been no appreciable profile development of 2 since the original topsoil on mass 4 was buried.

(d) Intermittent movement of mass 4 along failure surface 3 exposed unweathered sand and gravel subsoil along the line A-B. As movement took place, concurrent erosion by surface run-off and wash deposited the eroded materials downslope at area 5. Organic layers in 5 are truncated along failure plane 3 and mark former ground surfaces that were exposed for sufficient time to permit vegetative growth.

(e) Vegetation in a nearby slide area, similar in topographic setting to location 6, Fig. 3, consists of trees that range up to 60 years in age.

Pioneer inhabitants living near the slide areas say that vertical cracks have opened up along the upland in the last 50 years and that noticeable amounts of subsidence have been detected on failed masses nearest the upland. Railroads cross a number of slump areas that show signs of recent movement in the topography. Maintenance crews in these areas report the need for continual horizontal and vertical realignment (Fig. 4).

In those slump areas which appear to be creeping imperceptibly slowly and intermittently at present, little if any topsoil and only sparse grass vegetation have formed on failure surfaces; ridges are characteristically sharp and slope changes are abrupt and unsubdued; gully beds are frequently ungraded, i.e., there are abrupt drops at fault planes. For the most part, failed valley slopes commonly range from 1:5 to 1:15, however, seldom show unnatural shorelines (i.e., bulges at the toe of failed slopes).

## Dykes

A study has been made of the dykes in the R-1 area (Fig. 1 and page 9, Peterson's paper). It would appear they have been formed by surface run-off and gravity removing materials from the beds of small, steep-gradient draws located on opposite sides of the depression at location 6, Fig. 3, the depression having developed when nearby horizontal movement of the failing valley slope took place. Since the draws, or gullies, contributing the dyke-forming materials are directly opposite each other rather than staggered, it is postulated that the vertical crack formed by the horizontal slope movement cut through gully beds that existed prior to initial movement. Concurrent riverward creep of the ridge and valley side downslope from the dykes, subsidence

of the block that the dykes rest on and present erosion on the sides of the dykes have together produced the dykelike form. The material composing these small forms agrees with this origin. The well graded material in the dykes is due to contributions from the clean, unweathered sand and gravel subsoil and from clay, silt and organic matter from the weathered soil surface on the upland and ridge summit. The organic bands in the dykes have the same origin as those mentioned in (d), above, and shown in Fig. 3. Thus the dykes, having the origin ascribed to them above, would be postglacial as well as post slope-movement in age (which is very recent since this failure area is next to the undisturbed upland) rather than having formed before the last ice retreat as Peterson suggests (P. 4, L. 41).

## CONCLUSIONS

The engineering implications suggested by recurrent geologic and topographic features at the slide areas investigated are important in planning construction works on these valley slopes. The following conclusions are thought to carry with them important engineering implications:

(1) The slide topography occurs consistently on slopes adjoining places of recent active lateral or vertical stream erosion; in post-glacially eroded valleys whose present beds are located below the buried, glacial drift-covered surface of soft and weathered Bearpaw clay-shales.

In places the slope on the surface of the soft shale is very slight (e.g., diagram B, Fig. 1).

(2) Slope movement in the slide areas studied is postglacial in age and in some areas of "fresh looking" slump topography, slopes are creeping slowly and over a large area at the present time. The best available estimates place the average rate of this large-scale creep at something less than 1 foot per decade, thus corresponding to Terzaghi's definition of creep (3). Reports on three active slide areas along the South Saskatchewan River, however, some distance from the site of Peterson's investigations, state that in local areas movement rates average about 6 inches per year.

(3) The repeated failure of slopes ranging from 1:5 to 1:15 has in many places produced intricately deformed topography. Because of the difference between quick shear test strength values and stresses causing creep on flat slopes in the field, it would seem that shear stresses producing the large-scale creep must be only a small fraction of the values derived from shear tests of short duration.

In this connection, slopes on the surface of the shale are considerably steeper below river level. Shale slopes below the present river level must have been exposed for a time prior to valley alluviation; yet several profiles of the shale surface, extending across the valley in the damsite area, do not indicate the extensive slumping below river level that is so apparent above river level. This situation may possibly be ascribed to the behavior of the Bearpaw shales under longtime stressing and perhaps also to rebound following glacial preconsolidation. The importance of this is stated by Peterson on page 2, line 2: "While the harder shale would be stable on relatively steep slopes for a short period, much flatter slopes would be necessary for long time stability." As material presented in this discussion suggests, it is indeed difficult to predict the longtime behavior and safety factor of this unusual material, especially where loading conditions and natural slopes are likely to be altered by construction and where water under pressure is

transmitted to the surface of the soft shale. It would be instructive to learn the results of the program involving creep strength and longtime consolidation testing referred to by Peterson on page 14, line 3.

## REFERENCES


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2. Mollard, J. D., " Aerial Photographic Studies on the Central Saskatchewan Irrigation Project," Doctor's thesis, Cornell University, February, 1952.
3. Terzaghi, K.," Mechanism of Landslides," from "Application of Geology to Engineering Practice," Berkey Volume, Geological Society of America, 1950.

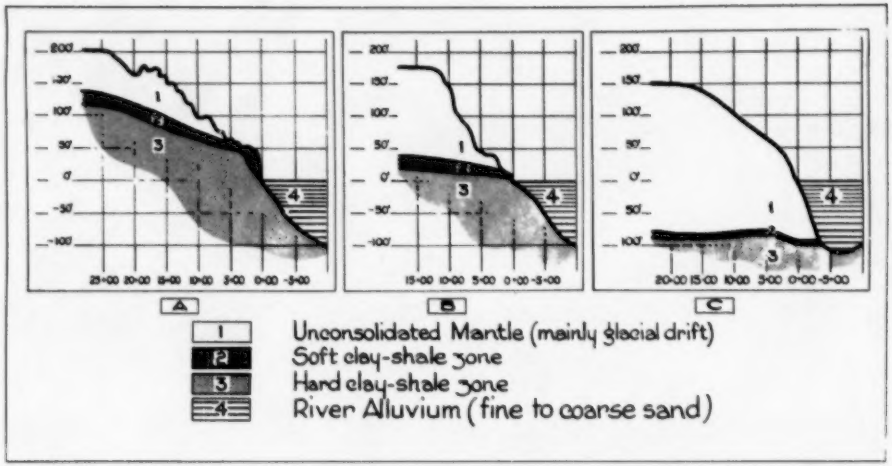


**Fig. 1. Three diagrams showing varying propensity of valley sides to fail by large-scale creep under different arrangements of slope-forming materials. Diagram A represents the least stable slope condition; C the most stable condition (profiles taken from drillhole data).**

Fig. 2. An aerial mosaic (see scale) showing five major slide areas (A, B, C, D, E) along the South Saskatchewan River, immediately downstream from the proposed damsite. Notice that the slide areas correspond with reaches of lateral river erosion (see arrows). The five slide areas seen in this mosaic have the following in common: (1) steep banks at the river's edge; (2) lack flood-plain development; (3) only a small amount of sandbar accretion in the river adjacent slides; (4) absence of obviously unnatural shorelines, indicative of recent and comparatively rapid slope movement.

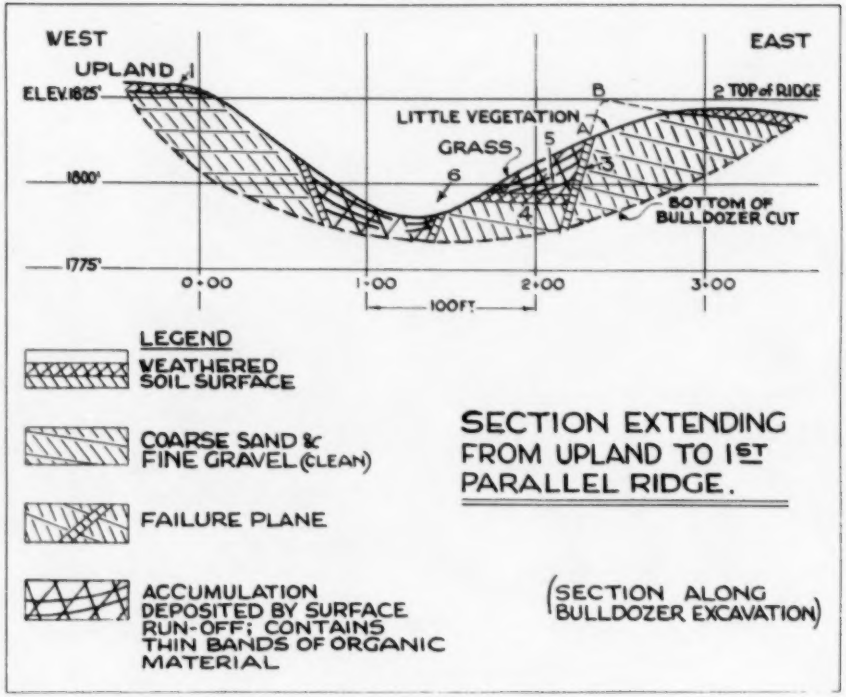


Fig. 3. Profile along bulldozer excavation extending from undisturbed upland to first slump ridge paralleling the upland in the R-1 area (Fig. 2, Peterson).

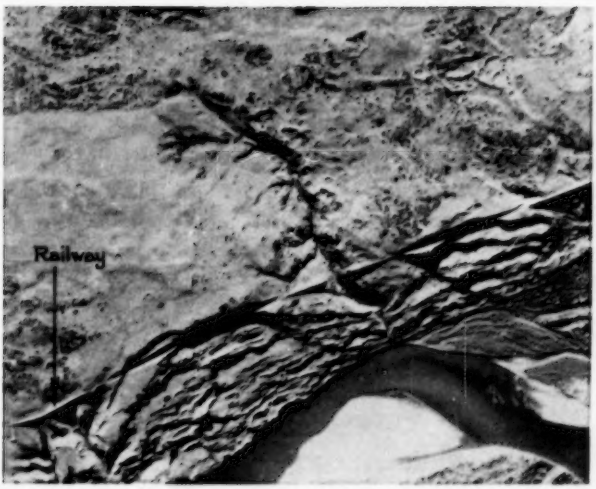


Fig. 4. An Aerial view showing a railway line constructed on "fresh looking" slide topography. Continual track realignment is needed along this section of valley side. Scale: 1 inch = 1320 feet.

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c. Discussion of several papers, grouped by Divisions.
d. Presented at the Atlanta (Ga.) Convention of the Society in February, 1954.
e. Presented at the Atlantic City (N.J.) Convention in June, 1954.

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